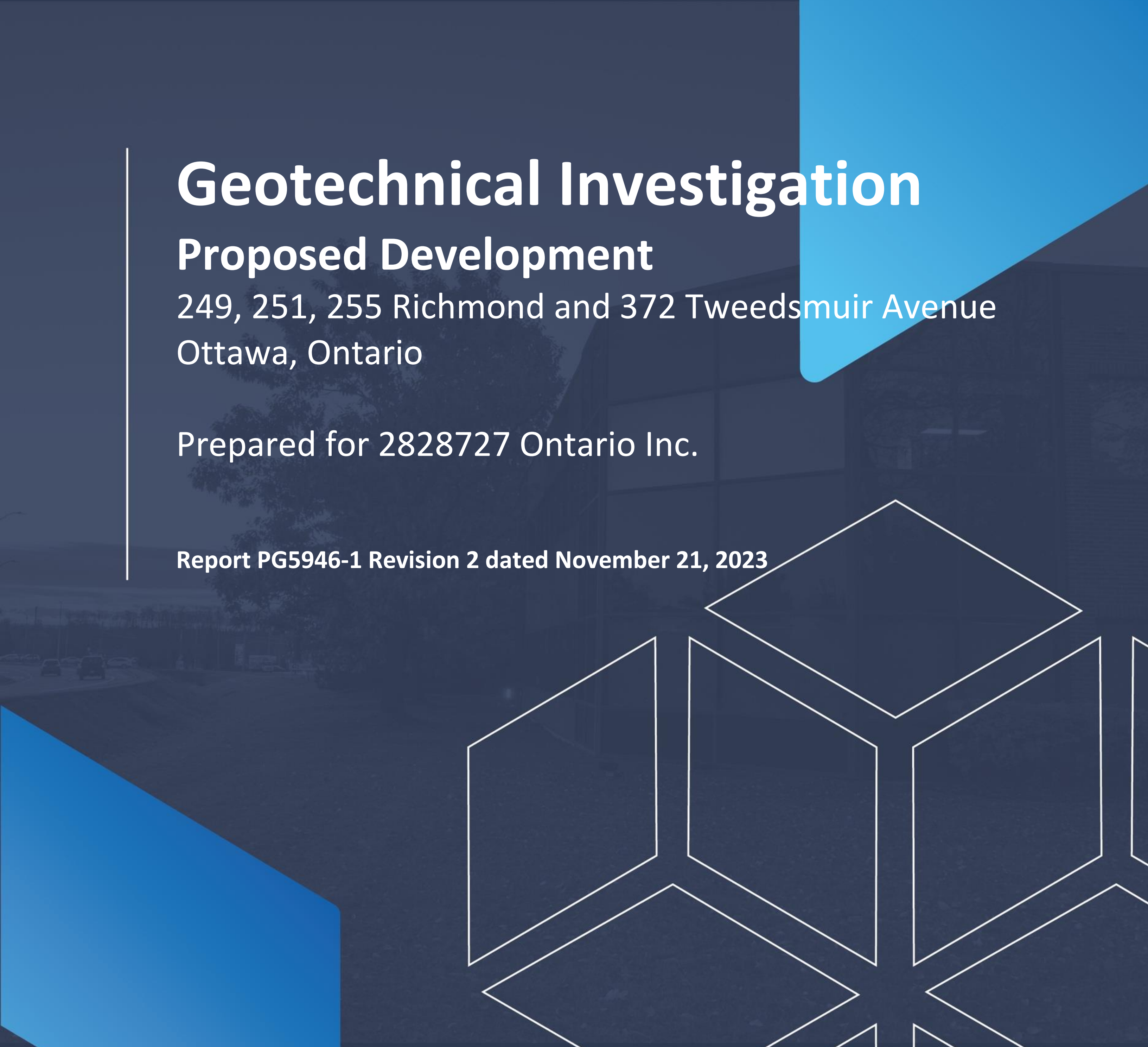

Geotechnical Investigation

Proposed Development

249, 251, 255 Richmond and 372 Tweedsmuir Avenue
Ottawa, Ontario

Prepared for 2828727 Ontario Inc.

Report PG5946-1 Revision 2 dated November 21, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by 2828727 Ontario Inc. to conduct a geotechnical investigation for the proposed development at 249, 251, 255 Richmond and 372 Tweedsmuir Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of boreholes.

- Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a 9-storey building with three underground parking levels to cover the majority of the site. Associated access lanes, hardscaped areas, and walkways are also anticipated as part of the development. It is expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on August 6, 2021 and consisted of advancing a total of three (3) boreholes to a maximum depth of 7.7 m below existing ground surface. A supplemental program was completed on subject site on July 22, 2022. At that time, two additional boreholes were advanced down to a maximum depth of 7.7m below existing ground surface. Previous geotechnical investigations were conducted by others in 2017. At that time, a total of seven boreholes were advanced to a maximum depth of 10.7 m below existing ground surface at the aforementioned site. The current test holes locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG5946-1 - Test Hole Location Plan included in Appendix 2.

The test holes were completed using a low clearance drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of drilling to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger and split-spoon were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Rock samples were recovered from borehole BH 1-21 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes, and transported to Paterson’s laboratory.

The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1
The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

A monitoring wells were installed in BH 1-21 and BH 4-22 to permit monitoring of the groundwater level subsequent to the completion of the sampling program.

Groundwater level measurements and observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5946-1- Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and ph. The laboratory test results are shown in Appendix 1 and are discussed in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is relatively flat and is currently occupied by several commercial and one residential building. A two-storey commercial building occupies the southeast portion, a one-storey commercial building occupies the central and west portion, and a single-family residential dwelling with associated driveway and backyard occupies the northern portion of the site. At-grade asphaltic concrete parking occupies the remaining southwestern portion of the site.

The site is bordered by two residential buildings to the north, Tweedsmuir Avenue followed by a one storey commercial building to the east, Richmond Road followed by a gas station to the south and by a low-rise commercial building to the west.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of an asphaltic concrete and fill layer followed by dense to very dense glacial till deposit overlying bedrock. The fill material consists of brown silty sand or clay with crushed stone, and trace coal and wood. The glacial till deposit encountered below the fill was observed to consist of compact to very dense brown silty sand with gravel, cobbles and boulders. Refusal to augering was encountered in BH 2-21, BH 3-21 and BH 5-22 at an approximate depth of 2.2 m. Bedrock was cored in BH 1-21 and BH 4-22 at a depth of 2.9m and 2.6m below ground surface, respectively, with an average RQD value ranging from 85 to 100%. This is indicative of a good to excellent quality bedrock at the location of the drilled boreholes.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Based on available geological mapping and augur refusal/coring, the bedrock in the subject area consists of interbedded limestone and dolomite of the Gull River formation, with an overburden drift thickness of 2 to 3 m depth.

4.3 Groundwater

Groundwater levels were measured during the current investigation on July 29, 2022 and the previous investigation on August 26, 2021, within the new and existing monitoring wells. The measured groundwater levels are presented in Table 1 below. The long-term groundwater table is expected to range between 5.0 to 10.0 m below existing grade.

<table>
<tr><th colspan="5">Table 1 – Summary of Groundwater Levels</th></tr>
<tr><th colspan="5">Current Investigation</th></tr>
<tr><th rowspan="2">Test Hole Number</th><th rowspan="2">Ground Surface Elevation (m)</th><th colspan="2">Measured Groundwater Level</th><th rowspan="2">Dated Recorded</th></tr>
<tr><th>Depth (m)</th><th>Elevation (m)</th></tr>
<tr><td>BH 1-21</td><td>66.76</td><td>6.22</td><td>60.54</td><td rowspan="7">July 29, 2022</td></tr>
<tr><td>BH 4-22</td><td>65.66</td><td>5.45</td><td>60.21</td></tr>
<tr><td colspan="4">Existing Monitoring wells – by others</td></tr>
<tr><td>MW 17-1</td><td>67.79</td><td>6.01</td><td>61.78</td></tr>
<tr><td>MW 17-2</td><td>67.62</td><td>6.99</td><td>60.63</td></tr>
<tr><td>MW 17-3</td><td>67.88</td><td>7.00</td><td>60.88</td></tr>
<tr><td>MW 17-4</td><td>67.35</td><td>9.92</td><td>57.43</td></tr>
<tr><td colspan="5">Note: The ground surface elevation at the borehole location was surveyed using a handheld GPS using a geodetic datum. Elevations for monitoring wells were provided by others.</td></tr>
</table>

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. Due to the presence of three levels of underground parking, it is anticipated that conventional shallow foundations placed on the clean surface sounded bedrock bearing surface will be suitable to support the proposed buildings.

It is anticipated that bedrock removal will be required to complete the underground parking levels of the proposed building. Line drilling and controlled blasting are recommended where large quantities of bedrock need to be removed. The blasting operations should be planned and carried out under the guidance of a professional engineer with experience in blasting operations.

Based on the anticipated excavation depth and the nature of the overburden, a temporary excavation support will be required along the upper portion of the excavation of the subject site. The design of the temporary shoring system needs to adequately support the existing building along the west side of the site, which is in close proximity with the proposed excavation.

In addition, due to the existence of 1200mm storm trunk sewer along Tweedsmuir Avenue, additional precautions should be taken during excavation activities to ensure that the existing service is not affected.

The above and other considerations are discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Due to the anticipated founding level for the proposed building, all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal will be required for the majority of the site for construction of the parking garage levels.

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. If encountered, existing foundation walls and other construction debris should be entirely removed from within the building perimeters.

Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below the final grade.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only a small quantity of bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings, and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in the proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries or claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing surrounding structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using near vertical sidewalls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing. The 1 m horizontal ledge set back can be eliminated with a shoring program which has drilled piles extending below the proposed founding elevation.

Vibration Considerations

Construction operations could cause vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles or sheet piling will require these pieces of equipment. Vibrations caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

These guidelines are for current construction standards. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Bedrock Excavation Face Reinforcement

A bedrock stabilization system consisting of a combination of horizontal rock anchors and/or chain link fencing connected to the excavation face may be required at specific locations to prevent bedrock pop-outs. This system is usually considered where bedrock fractures are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. This material should be used structurally only to build up the subgrade for pavements. Where the fill is open-graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded bedrock surface can be designed using a bearing resistance value at ultimate limits states (ULS) of **3,000 kPa**. A geotechnical factor of 0.5 was applied to the above noted bearing resistance value.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures, or open joints which can be detected from surface sounding with a rock hammer.

Settlement

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for foundations constructed at the subject site. A higher site class, such as Class A or B may be provided for foundations placed on or near the bedrock surface. However, the higher site class will need to be confirmed by a site-specific seismic shear wave velocity test.

The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the bedrock surface will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor. Pipe spacing requirements should be determined at the time of excavation when the groundwater infiltration can be better assessed.

An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. The upper 200 mm below the basement floor slab should consist of a 19 mm clear crushed stone. Alternatively, excavated limestone bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the Paterson at the time of placement.

In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor. Pipe spacing requirements should be determined at the time of excavation when the groundwater infiltration can be better assessed

5.6 Basement Wall

It is understood that the basement walls are to be poured against a water suppression system, which will be placed against the temporary shoring system and/or exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m3 (effective 15.5 kN/m3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slab, which should be designed to accommodate these pressures. A hydrostatic pressure should be added for the portion below groundwater level.

Where the soil is to be retained, there are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $Ko \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45-a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.281 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using

$$P_o = 0.5\ K_o\ \gamma\ H^2$$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o\cdot(H/3)+\Delta P_{AE}\cdot(0.6\cdot H)\}/P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in limestone bedrock is based upon two possible failure modes. The rock anchor can fail either by shear failure along the grout/rock interface or by pullout at 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the individual anchor load capacity.

A third failure mode of shear failure along the grout/steel interface should be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada) or Williams Form Engineering, have qualified personnel on staff to recommend appropriate rock anchor size and materials.

Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout fluid does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not, prior to servicing. To resist seismic uplift pressures, a passive rock anchor system is adequate. However, a post-tensioned anchor will absorb the uplift load pressure with less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor is provided with a fixed anchor length at the anchor base, which will provide the anchor capacity, and a free anchor length between the rock surface and the top of the bonded length.

As the depth at which the apex of the shear failure cone develops midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a sleeve to act as a bond break, with the sleeve filled with grout. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between 75 and 100 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, should be provided. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 72** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Grouted Rock Anchor Lengths

Parameters used to calculate grouted rock anchor lengths are provided in Table 2.

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	72 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone	75 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths are provided in Table 3. The factored tensile resistance values provided are based on a single anchor with no group influence effects.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.7	0.7	2.4	450
	2.2	0.7	2.9	600
	2.5	0.8	3.3	750
	2.8	1.0	3.8	900
125	1.0	1.0	2.0	450
	1.3	1.1	2.4	600
	1.6	1.2	2.8	750
	1.9	1.2	3.1	900

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by Paterson personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended. The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lower level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 below. The flexible pavement structure presented in Table 5 should be used for at grade access lanes and heavy loading parking areas overlying the podium deck.

Table 4 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE Fill or OPSS Granular B Type I or II material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 5 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type II overlying the Concrete Podium Deck.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

The upper 300 mm of the bedrock surface should be reviewed and approved by Paterson prior to placing the base and subbase materials. Care should be exercised to ensure that the bedrock subgrade does not have depressions that will trap water.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Water Suppression System and Foundation Drainage

It is understood that the proposed structure will occupy the entire boundary of the subject site. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be blind poured against a foundation drainage/waterproofing system placed directly against the temporary shoring system and a suitably prepared bedrock surface. It is suggested that this system be constructed as follows:

- Temporary shoring system and/or bedrock vertical faces should be prepared to receive a waterproofing membrane, such as lined bentonite sheets, and drainage board for the underground parking structure. The bedrock surface will be prepared by grinding bedrock face high points and in-filling bedrock face cavities or using shotcrete to smooth out angular sections depending on the manufacturer's requirements of the proposed waterproofing membrane.

- A waterproofing membrane will be applied to the temporary shoring system and prepared vertical bedrock surface from ground surface to the founding elevation. The waterproofing membrane should also be extended horizontally below the proposed footings a minimum of 600 mm away from the face of the excavation. The membrane will serve as a water infiltration suppression system. The membrane will also be placed along the horizontal surface beneath the perimeter footings to provide a better seal at the vertical and horizontal interface.

- A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the bottom of the foundation wall. It is expected that 150 mm diameter sleeves placed at 3 m centers be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration for the lower basement area. For preliminary design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at every bay opening. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided to the elevator pits (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

The footings located along parking garage entrance may require protection against frost action depending on the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. However, for most of the site, insufficient room will be available to permit the building excavation to be constructed by open-cut methods (i.e., unsupported excavations).

Unsupported Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

In bedrock, almost vertical side slopes can be used provided that all loose rock and blocks with unfavorable weak planes are removed or stabilized.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system.

The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

The temporary shoring system could consist of a soldier pile and lagging system or steel sheet piles. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method. The earth pressures acting on the temporary shoring system may be calculated with the following parameters.

Table 6 – Soils Parameter for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_O)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Underpinning of Adjacent Structures

Founding conditions of the adjacent structure along the west boundary of the proposed building should be assessed and underpinning requirements should be evaluated.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. At least 300 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) glacial till above the cover material if the excavation and filling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement.

Well fractured bedrock should be acceptable as backfill for the lower portion of the trenches when the excavation is within bedrock provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones are 300 mm or smaller in their longest dimension.

The backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

Long-term Groundwater Control

Any groundwater encountered along the buildings' perimeter or sub-slab drainage system will be directed to the proposed buildings' cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, the expected long-term groundwater flow should be low (i.e., less than 25,000 L/day/building) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The long-term groundwater flow is anticipated to be controllable using conventional open sumps.

Impacts on Neighbouring Properties

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures.

Due to the proposed waterproofing to be installed along the perimeter of the proposed building, no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a non-aggressive to a slightly aggressive corrosive environment.

6.8 Impacts on the Existing Underground Service and Monitoring Program

Based on our review of the infrastructure service plan provided by GeoOttawa, it is understood that a 1200mm concrete storm sewer trunk is located along Tweedsmuir Avenue. It is further understood that the minimum clear horizontal spacing between the exterior face of the trunk and the property limit is 6.9m. However, the invert level of the existing storm trunk is not known.

Based on the available soil information, bedrock is anticipated to be located at an approximate depth of 3.0m below average ground surface. It is anticipated that the overburden material will be retained against a temporary shoring system to provide a vertical excavation face. In this case, the location of any proposed tiebacks should be reviewed in order to avoid interference with the existing storm sewer. However, in case an open cut excavation is permitted, then the sides of the excavation can be cut at a slope of 1H:1V in the overburden material. For the worst-case scenario of excavating the overburden material down to bedrock, a maximum 3m horizontal encroachment into the sidewalk on Tweedsmuir Avenue will be required.

Based on the available minimum horizontal setback of 6.9m between the property limit and the existing storm trunk, and on the available bedrock depth, the lateral support zone of 1.5H:1V in overburden and 1H:6V in good quality bedrock for the existing storm sewer pipe will be protected, irrespective of the pipe invert level, and the excavation of the proposed building will not negatively impact the existing storm pipe. None the less, a vibration monitoring plan should be implemented to ensure the vibrations during construction are kept within acceptable limits.

Highly weathered Bedrock

It is common to encountered highly weathered bedrock along the upper 1 to 2 m of the bedrock surface. Therefore, it is highly recommended that Paterson complete periodic inspections of the bedrock conditions once exposed. These areas are anticipated to require bedrock stabilization measures such as shotcrete, chain link fencing in conjunction with rock anchors, and/or full shoring system.

Vibration Monitoring Equipment

Vibration levels at the trunk sewer should be continuously monitored during the excavation and blasting programs. The vibration monitoring equipment should consist of 2 tri-axial seismographs, capable of measuring vibration intensities up to 254 mm/s at a frequency response of 2 to 250 Hz, and should be installed directly on top of the trunk sewer.

Vibration Monitoring Criteria

Paterson provided generalized vibration criteria for the 1,200 mm sewer trunk in the Geotechnical Investigation Report, which recommended a maximum vibration of up to 50 mm/s for frequencies exceeding 40 Hz, this is shown on Figure 1 below.

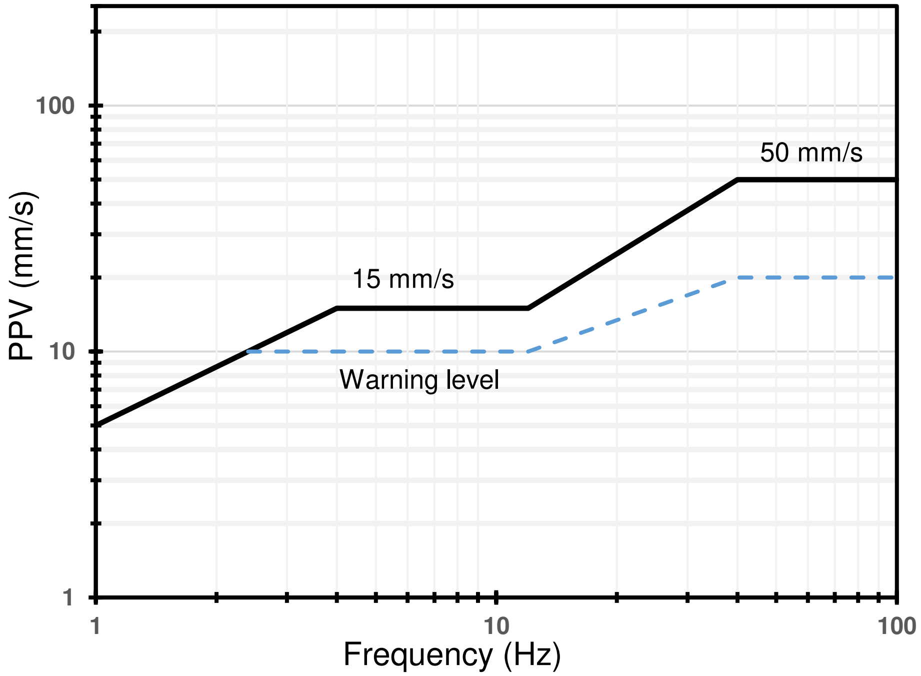


Figure 1 – Vibration Criteria for the Storm Trunk Sewer

If the recommended vibration limit is exceeded, Paterson will notify the site superintendent and operation will be stopped, and the vibration monitoring points on the trunk sewer will be surveyed. Weekly reporting of the monitoring program and recommendations will be provided to the owner and the City of Ottawa.

The monitoring protocol should include the following information:

Warning Level Event

Paterson will review all vibrations over the established warning level, illustrated by the blue line in the above figure, and; Paterson will notify the contractor if any vibrations occur due to construction activities and are close to exceedance level.

Exceedance Level Event

Paterson will notify all the relevant stakeholders via email if any vibrations surpass the exceedance level, illustrated by the black line in the above figure.

Paterson will also ensure the monitors are still functioning and will issue the vibration exceedance result. The data collected should include the following:

☐ Measured vibration levels
☐ Distance from the construction activity to monitoring location.
☐ Vibration type

Monitoring should be compliant with all related regulations.

Incidence & Exceedance Reporting

In case a vibration incident/exceedance occurs from construction activities, the Senior Project Management and any relevant personnel should be notified immediately. A report should be completed which contains the following:

☐ Identify the location of vibration exceedance.
☐ The date, time and nature of the exceedance/incident
☐ Purpose of the exceeded monitor and current vibration criteria
☐ Identify the likely cause of the exceedance/incident.
☐ Describe the response action that has been completed to date.
☐ Describe the proposed measures to address the exceedance/incident.

The contractor should implement mitigation measures for future excavation or any construction activities as necessary and provide updates on the effectiveness of the improvement. Response actions should be pre-determined prior to excavation, depending on the approach provided to protect elements. Processes and procedures should be in-place prior to completing any vibrations to identify issues and react in a quick manner in the event of an exceedance.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

- Review of the geotechnical aspects of the excavating program, prior to construction.
- Observation of all bearing surfaces prior to the placement of concrete.
- Inspection of the perimeter and underfloor drainage system, and the waterproofing of basement walls and elevator pit.
- Sampling and testing of the concrete and fill materials.
- Vibration monitoring and geophone installation.
- Vibration action plan and design, if requested.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than 2828727 Ontario Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Nov. 21, 2023
M. SALEH
100507739
PROVINCE OF ONTARIO

Zubaida Al-Moselly, P.Eng.

Maha Saleh, P.Eng.

Report Distribution:

☐ 2828727 Ontario Inc (email copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

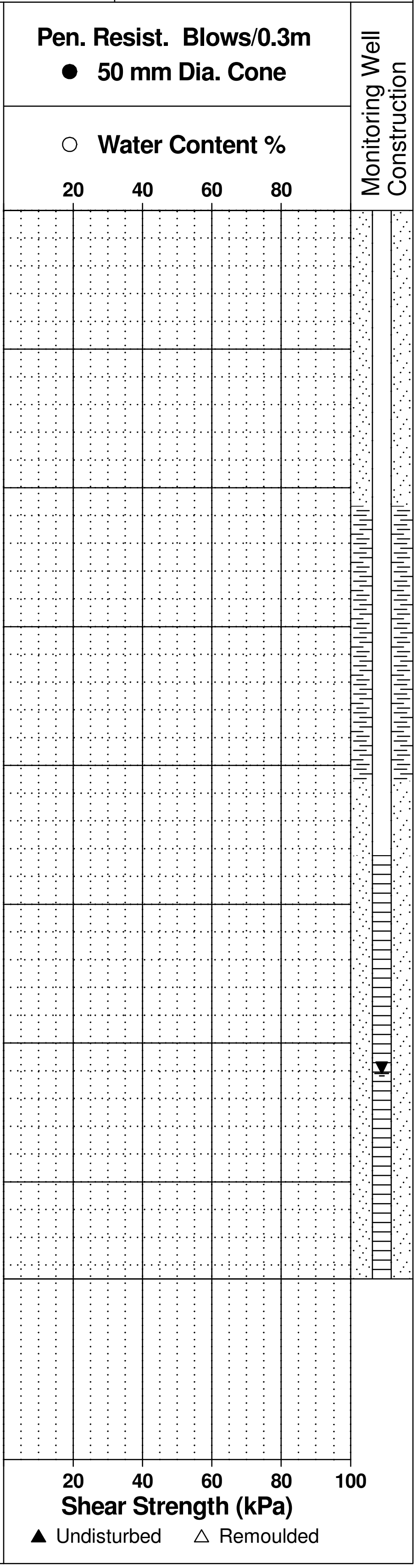
patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
249, 251, 255 Richmond Road & 372 Tweedsmuir Ave.
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE August 6, 2021
FILE NO. PG5946
HOLE NO. BH 1-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	66.76
Asphaltic concrete 0.13						
FILL: Brown silty sand with crushed stone 0.70	AU	1				
	SS	2	50	50+		
					1	65.76
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders, trace clay 2.92	SS	3	100	50+		
					2	64.76
	SS	4	94	50+		
BEDROCK: Good to excellent quality, grey silty dolostone interbedded with grey limestone 7.70	RC	1	100	100	3	63.76
	RC	2	97	85		
					4	62.76
					5	61.76
	RC	3	100	88		
					6	60.76
	RC	4	100	100		
					7	59.76
End of Borehole						
(GWL @ 6.22m - July 29, 2022)						



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
249, 251, 255 Richmond Road & 372 Tweedsmuir Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 6, 2021

FILE NO. PG5946

HOLE NO. BH 2-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	67.46		
Asphaltic concrete 0.10									
FILL: Brown silty sand with crushed stone 0.76		AU	1						
Compact, brown **SILTY SAND** with gravel 1.52		SS	2	42	12	1	66.46		
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders 2.23		SS	3	50	13	2	65.46		
End of Borehole Practical refusal to augering at 2.23m depth. (BH dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
249, 251, 255 Richmond Road & 372 Tweedsmuir Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 6, 2021

FILE NO. PG5946

HOLE NO. BH 3-21

SOIL DESCRIPTION		SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
GROUND SURFACE						0	66.83		
Asphaltic concrete 0.10									
FILL: Brown silty sand with crushed stone 0.76		AU	1						
Hard to very stiff, brown **SILTY CLAY,** trace gravel 1.68		SS	2	42	10	1	65.83		
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders 2.23		SS	3	42	35	2	64.83		
End of Borehole Practical refusal to augering at 2.23m depth. (BH dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
249, 251, 255 Richmond Road & 372 Tweedsmuir Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

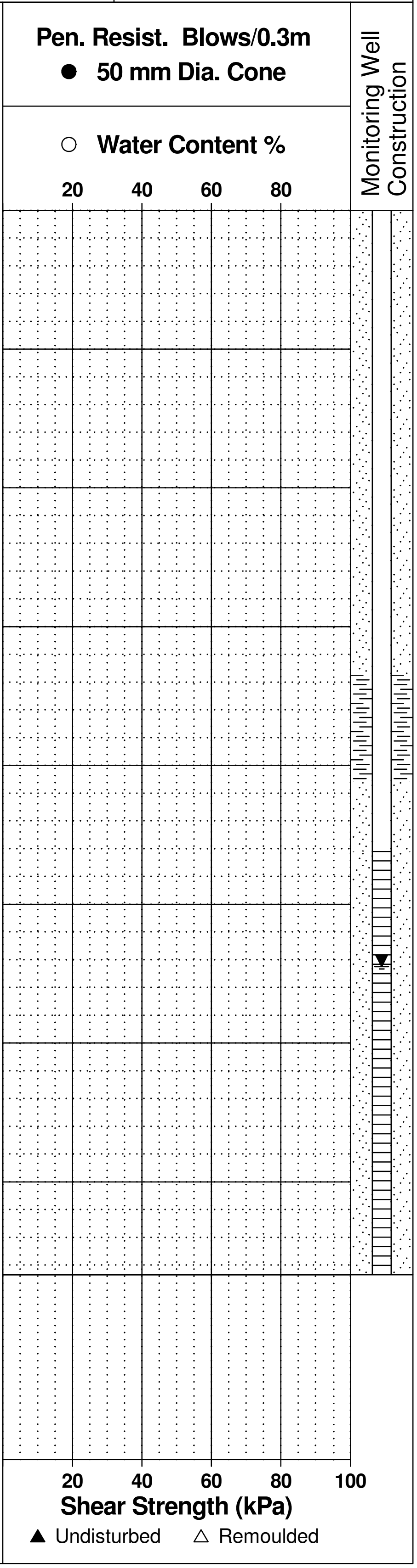
DATE July 22, 2022

FILE NO. PG5946

HOLE NO. BH 4-22

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.03				
FILL: Brown silty sand, some gravel, trace topsoil	0.46	AU	1		
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders		SS	2	67	43
		SS	3	0	50+
		RC	2	100	85
	2.64	RC	1	100	62
BEDROCK: Good to excellent quality, grey silty dolostone interbedded with grey limestone - 20mm thick mud seam at 4.4m depth		RC	3	100	88
		RC	4	100	94
	7.67	RC	5	100	100
End of Borehole (GWL @ 5.45m - July 29, 2022)					

DEPTH (m)	ELEV. (m)
0	65.66
1	64.66
2	63.66
3	62.66
4	61.66
5	60.66
6	59.66
7	58.66



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
249, 251, 255 Richmond Road & 372 Tweedsmuir Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 22, 2022

FILE NO. PG5946

HOLE NO. BH 5-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	65.49		
TOPSOIL 0.18		AU	1						
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders		SS	2	67	49	1	64.49		
		SS	3	75	50+				
2.19						2	63.49		
End of Borehole Practical refusal to augering at 2.19m depth.									

Shear Strength (kPa)

20	40	60	80	100

▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

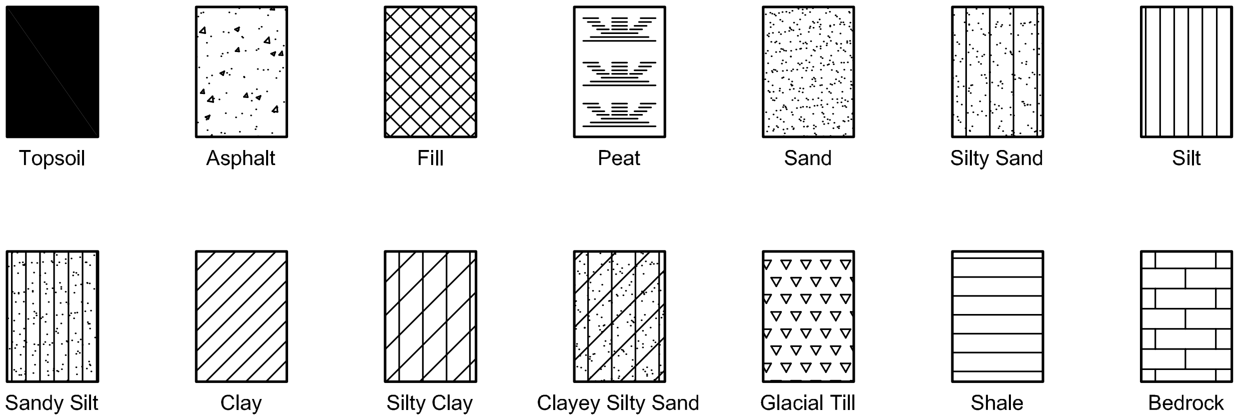
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

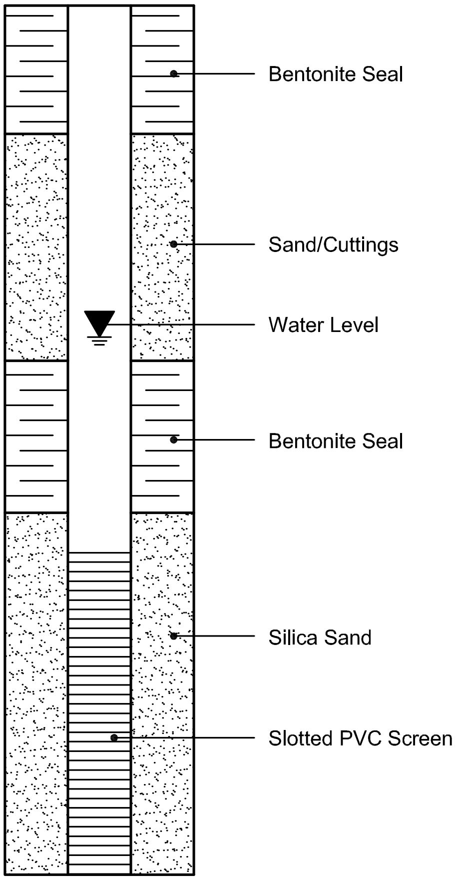
SYMBOLS AND TERMS (continued)

STRATA PLOT

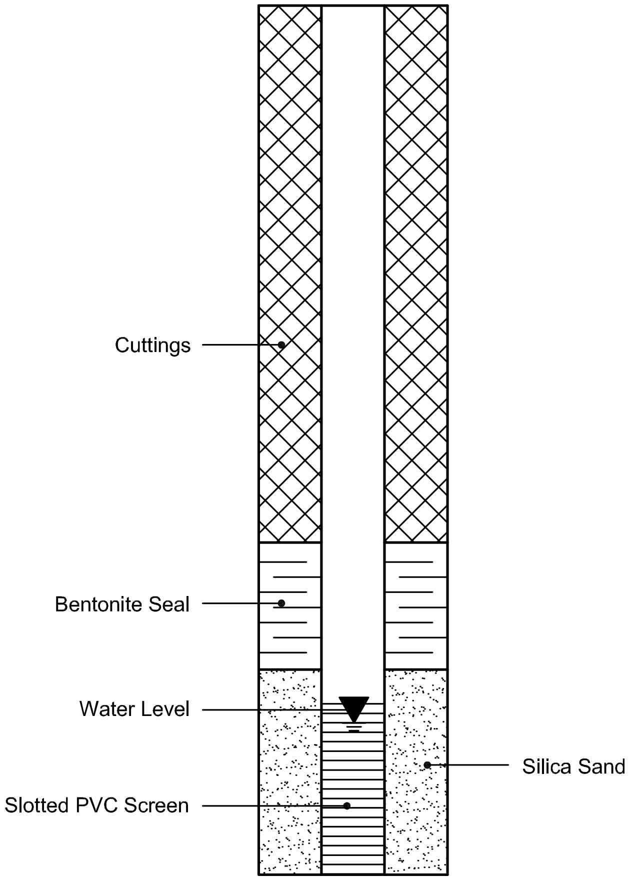


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Log of Borehole MW17-1

Project No: OTT-00241482-B0

Project: Supplemental Phase II ESA

Location: 255 Richmond Road, Ottawa, Ontairo

Figure No. 3

Page. 1 of 1

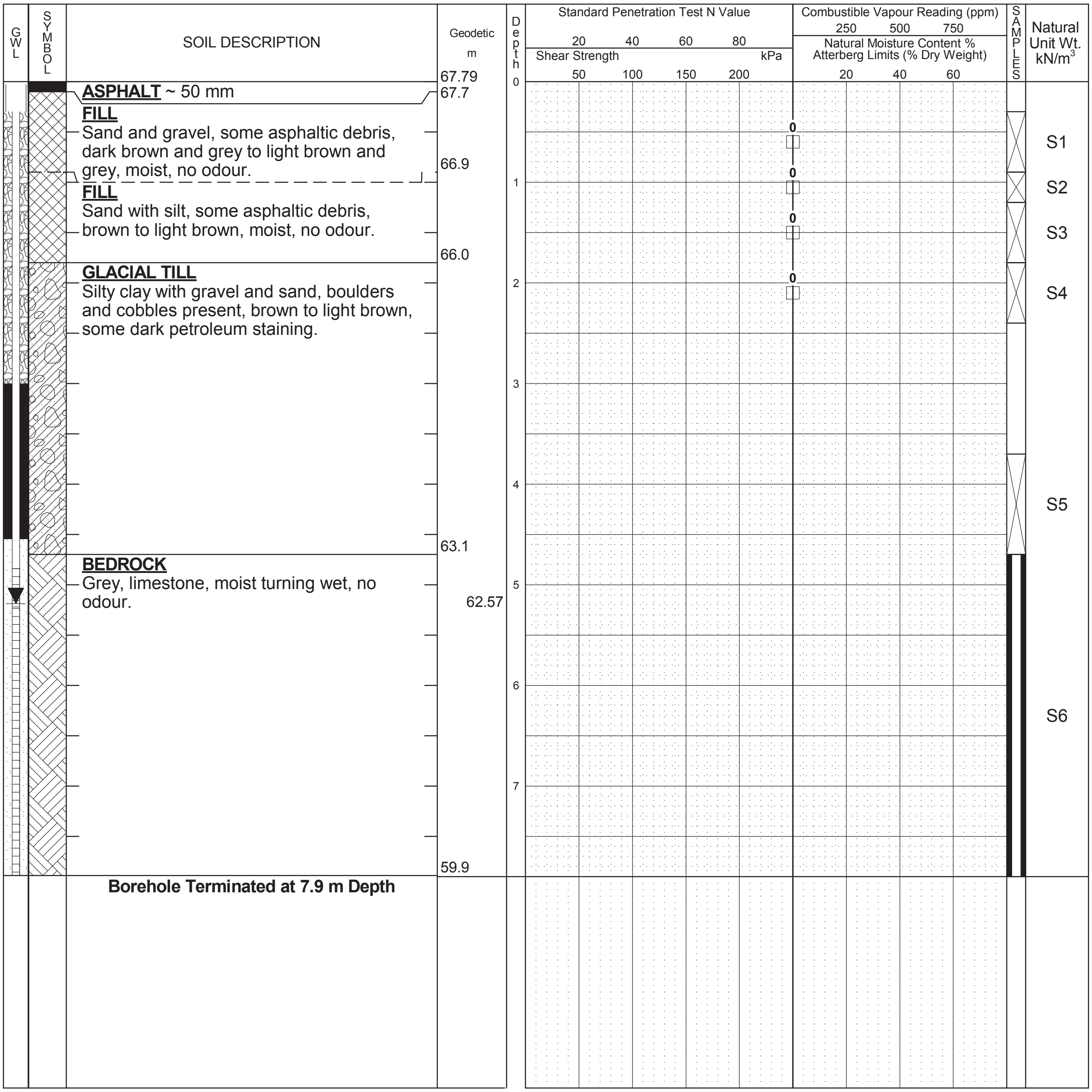
Date Drilled: August 4th, 2017

Drill Type: Geoprobe 420M

Datum: Geodetic

Logged by: JO Checked by: MGM

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



Geodetic m	Soil description
67.79	
67.7	**ASPHALT** ~ 50 mm
66.9	**FILL** Sand and gravel, some asphaltic debris, dark brown and grey to light brown and grey, moist, no odour.
66.0	**FILL** Sand with silt, some asphaltic debris, brown to light brown, moist, no odour.
63.1	**GLACIAL TILL** Silty clay with gravel and sand, boulders and cobbles present, brown to light brown, some dark petroleum staining.
62.57	
59.9	**BEDROCK** Grey, limestone, moist turning wet, no odour.

Borehole Terminated at 7.9 m Depth

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A flushmount monitoring well with a 32 mm slotted standpipe was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241482-B0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
6 days	7.1	
95 days	5.9	
102 days	5.2	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW17-2

Project No: OTT-00241482-B0

Project: Supplemental Phase II ESA

Location: 255 Richmond Road, Ottawa, Ontairo

Figure No. 4

Page. 1 of 1

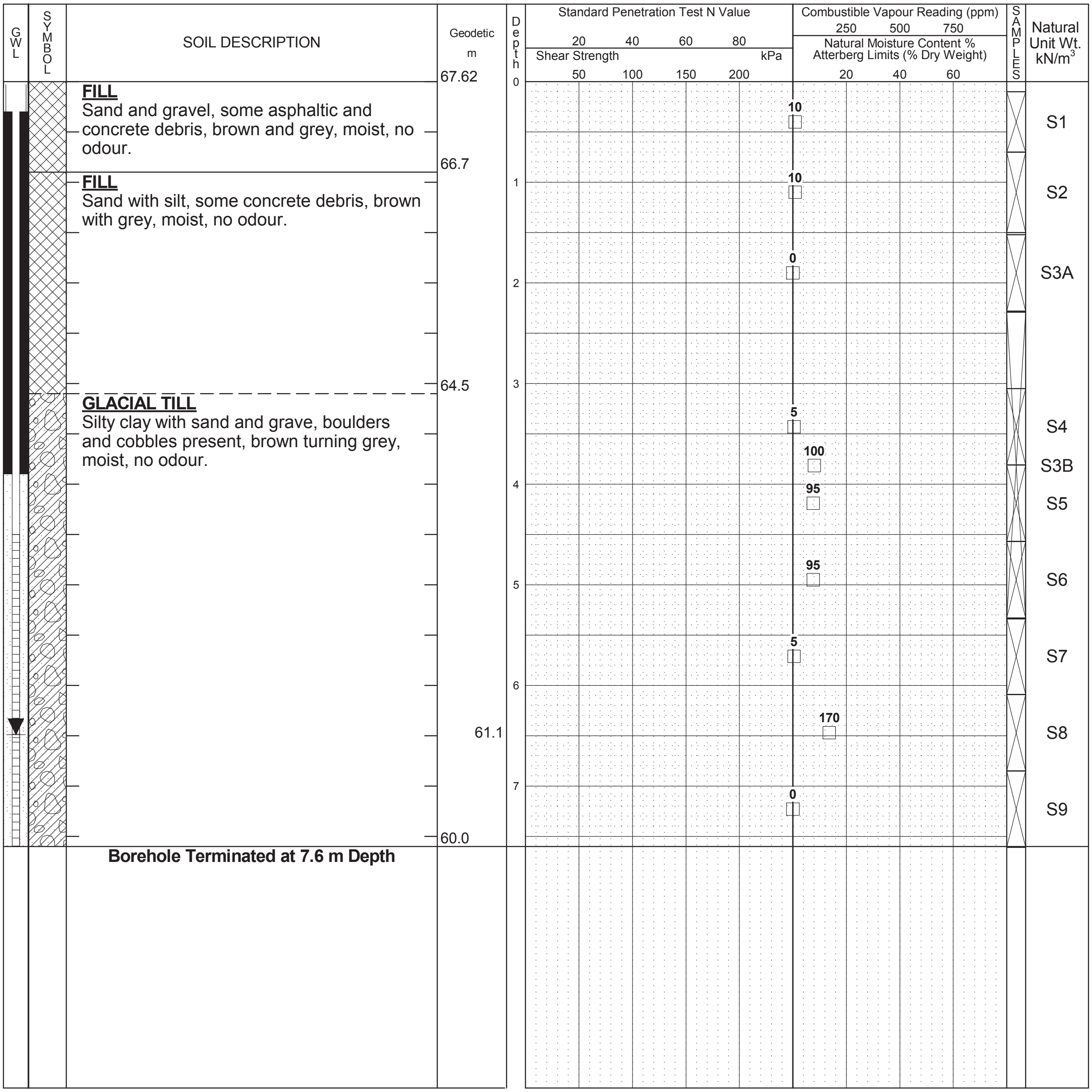
Date Drilled: October 16th, 2017

Drill Type: Geoprobe 420M

Datum: Geodetic

Logged by: JO Checked by: MGM

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



67.62

FILL
Sand and gravel, some asphaltic and concrete debris, brown and grey, moist, no odour.

66.7

FILL
Sand with silt, some concrete debris, brown with grey, moist, no odour.

64.5

GLACIAL TILL
Silty clay with sand and grave, boulders and cobbles present, brown turning grey, moist, no odour.

61.1

60.0

Borehole Terminated at 7.6 m Depth

Sample	Combustible Vapour Reading (ppm)
S1	10
S2	10
S3A	0
S4	5
S3B	100
S5	95
S6	95
S7	5
S8	170
S9	0

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A flushmount monitoring well with a 32 mm slotted standpipe was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241482-B0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
17 days	7.2	
24 days	6.5	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW17-3

Project No: OTT-00241482-B0

Project: Supplemental Phase II ESA

Location: 255 Richmond Road, Ottawa, Ontairo

Figure No. 5

Page. 1 of 1

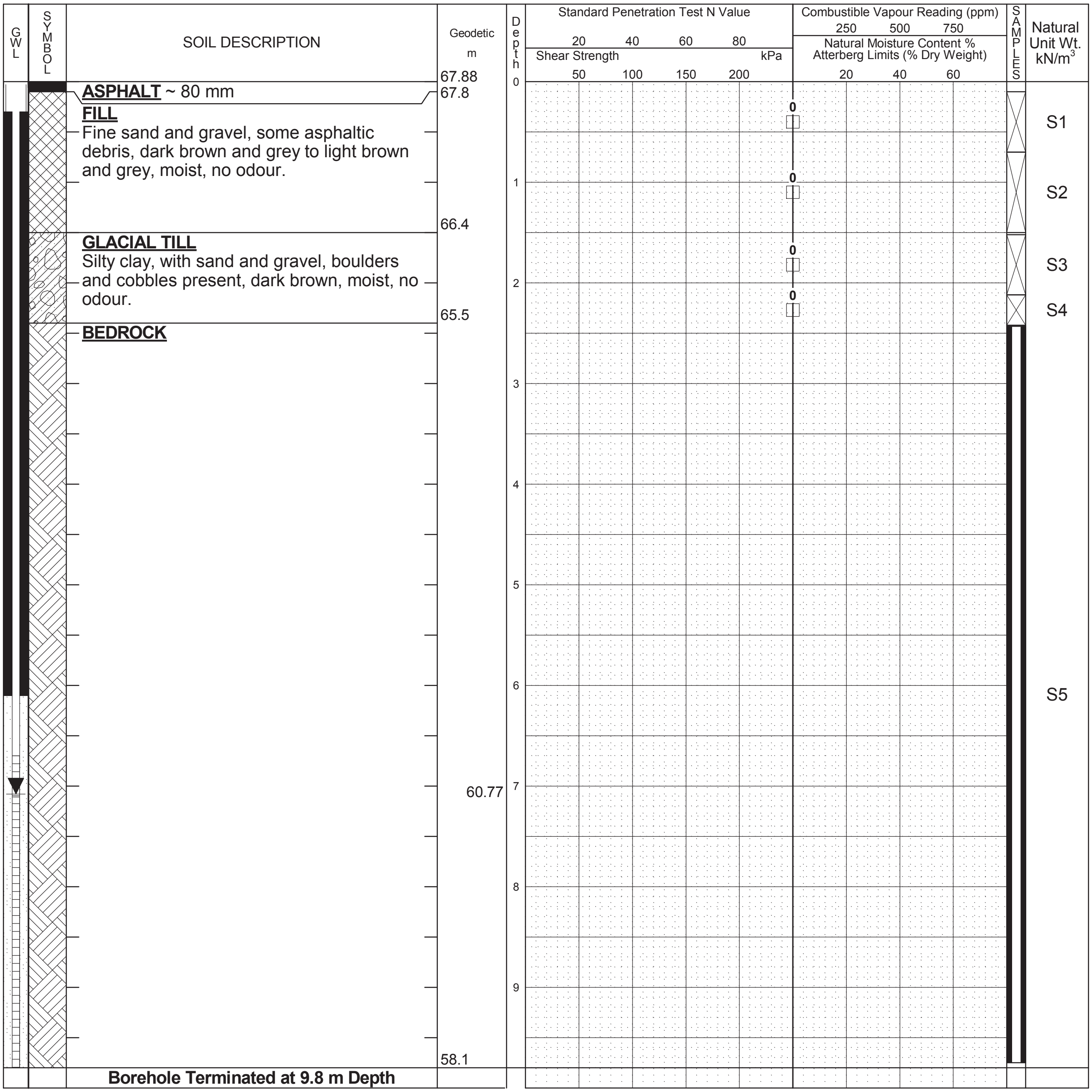
Date Drilled: October 16th, 2017

Drill Type: Geoprobe 420M

Datum: Geodetic

Logged by: JO   Checked by: MGM

Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



SOIL DESCRIPTION	Geodetic m	Depth	Natural Unit Wt. kN/m³
	67.88	0	
ASPHALT ~ 80 mm	67.8		
FILL Fine sand and gravel, some asphaltic debris, dark brown and grey to light brown and grey, moist, no odour.		1	S1 S2
GLACIAL TILL Silty clay, with sand and gravel, boulders and cobbles present, dark brown, moist, no odour.	66.4	2	S3 S4
BEDROCK	65.5	3 4 5 6	S5
	60.77	7 8 9	
	58.1		
Borehole Terminated at 9.8 m Depth			

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A flushmount monitoring well with a 32 mm slotted standpipe was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241482-B0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
17 days	6.6	
24 days	7.1	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW17-4

Project No: OTT-00241482-B0

Project: Supplemental Phase II ESA

Location: 255 Richmond Road, Ottawa, Ontairo

Figure No. 6

Page. 1 of 2

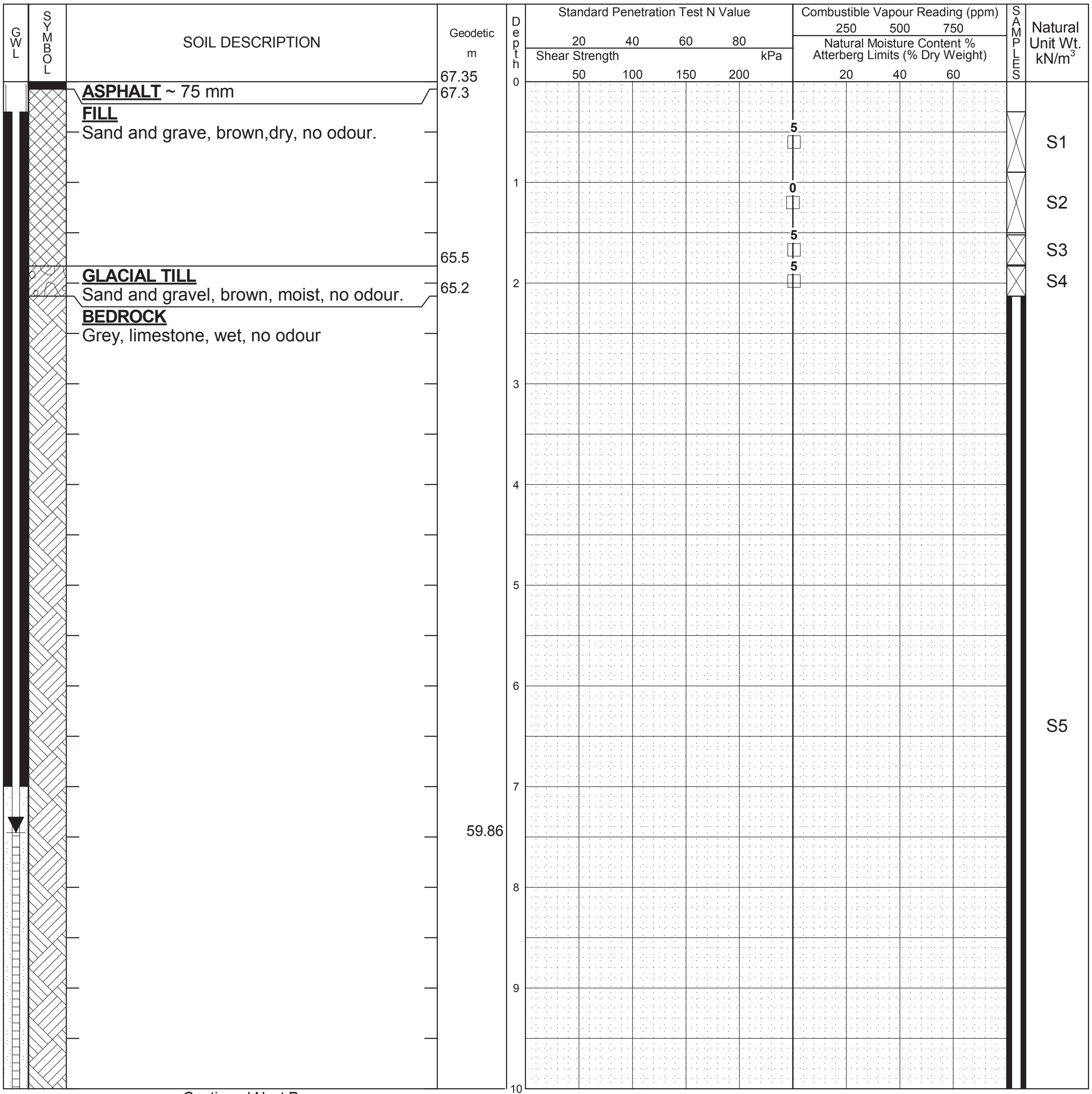
Date Drilled: October 16th, 2017

Drill Type: Geoprobe 420M

Datum: Geodetic

Logged by: JO Checked by: MGM

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢—○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



ASPHALT ~ 75 mm

FILL
Sand and grave, brown,dry, no odour.

GLACIAL TILL
Sand and gravel, brown, moist, no odour.

BEDROCK
Grey, limestone, wet, no odour

Continued Next Page

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A flushmount monitoring well with a 32 mm slotted standpipe was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241482-B0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
17 days	6.8	
24 days	7.5	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW17-4

Project No: OTT-00241482-B0

Project: Supplemental Phase II ESA

Figure No. 6

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value 20 40 60 80 Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 Natural Moisture Content % Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m³
			57.35	10				
		BEDROCK Grey, limestone, wet, no odour *(continued)*						
			56.7					
		Borehole Terminated at 10.67 m Depth						

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A flushmount monitoring well with a 32 mm slotted standpipe was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241482-B0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
17 days	6.8	
24 days	7.5	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW17-5

Project No: OTT-00241482-B0

Project: Supplemental Phase II ESA

Location: 255 Richmond Road, Ottawa, Ontairo

Figure No. 7

Page. 1 of 1

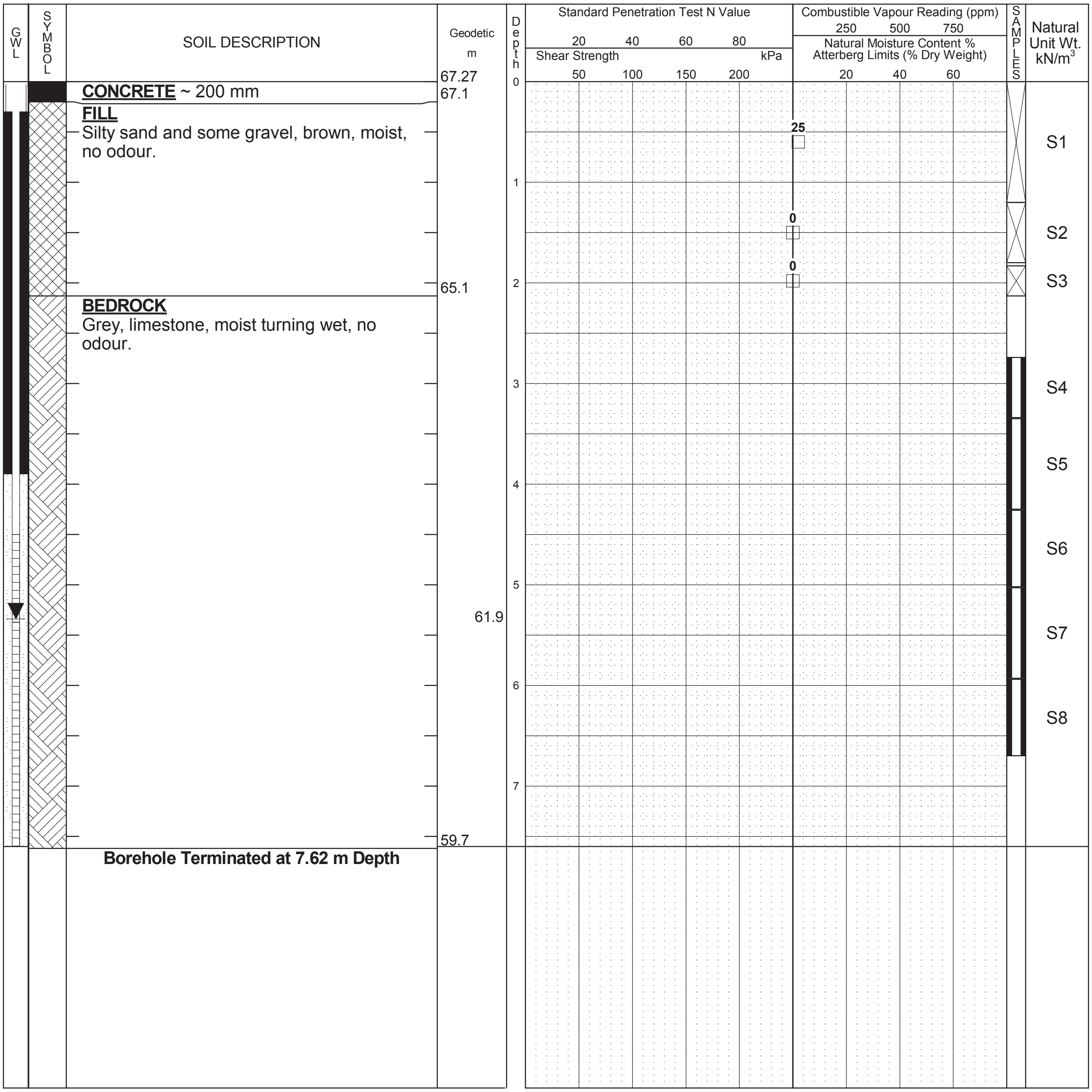
Date Drilled: October 23rd, 2017

Drill Type: Geoprobe 420M

Datum: Geodetic

Logged by: JO Checked by: MGM

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



CONCRETE ~ 200 mm

FILL
Silty sand and some gravel, brown, moist, no odour.

BEDROCK
Grey, limestone, moist turning wet, no odour.

Borehole Terminated at 7.62 m Depth

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A flushmount monitoring well with a 32 mm slotted standpipe was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241482-B0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
10 days	4.4	
17 days	5.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW17-6

Project No: OTT-00241482-B0

Figure No. 8

Page. 1 of 1

Project: Supplemental Phase II ESA

Location: 255 Richmond Road, Ottawa, Ontairo

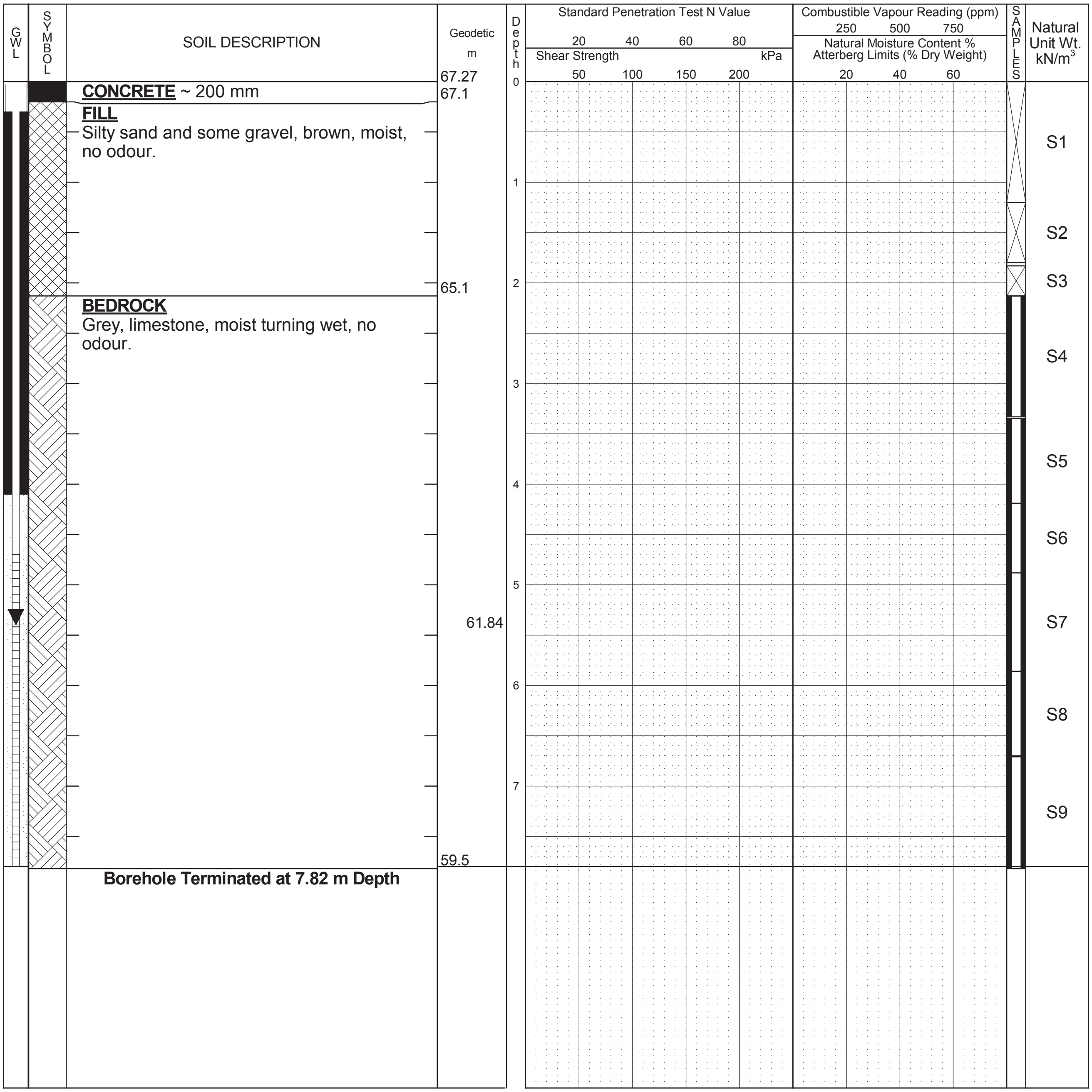
Date Drilled: October 23rd, 2017

Drill Type: Geoprobe 420M

Datum: Geodetic

Logged by: JO Checked by: MGM

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



SOIL DESCRIPTION	Geodetic m
	67.27
CONCRETE ~ 200 mm	67.1
FILL Silty sand and some gravel, brown, moist, no odour.	
BEDROCK Grey, limestone, moist turning wet, no odour.	65.1
	59.5
Borehole Terminated at 7.82 m Depth	

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A flushmount monitoring well with a 32 mm slotted standpipe was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241482-B0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
9 days	4.6	
18 days	5.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW17-7

Project No: OTT-00241482-B0

Project: Supplemental Phase II ESA

Location: 255 Richmond Road, Ottawa, Ontairo

Figure No. 9

Page. 1 of 1

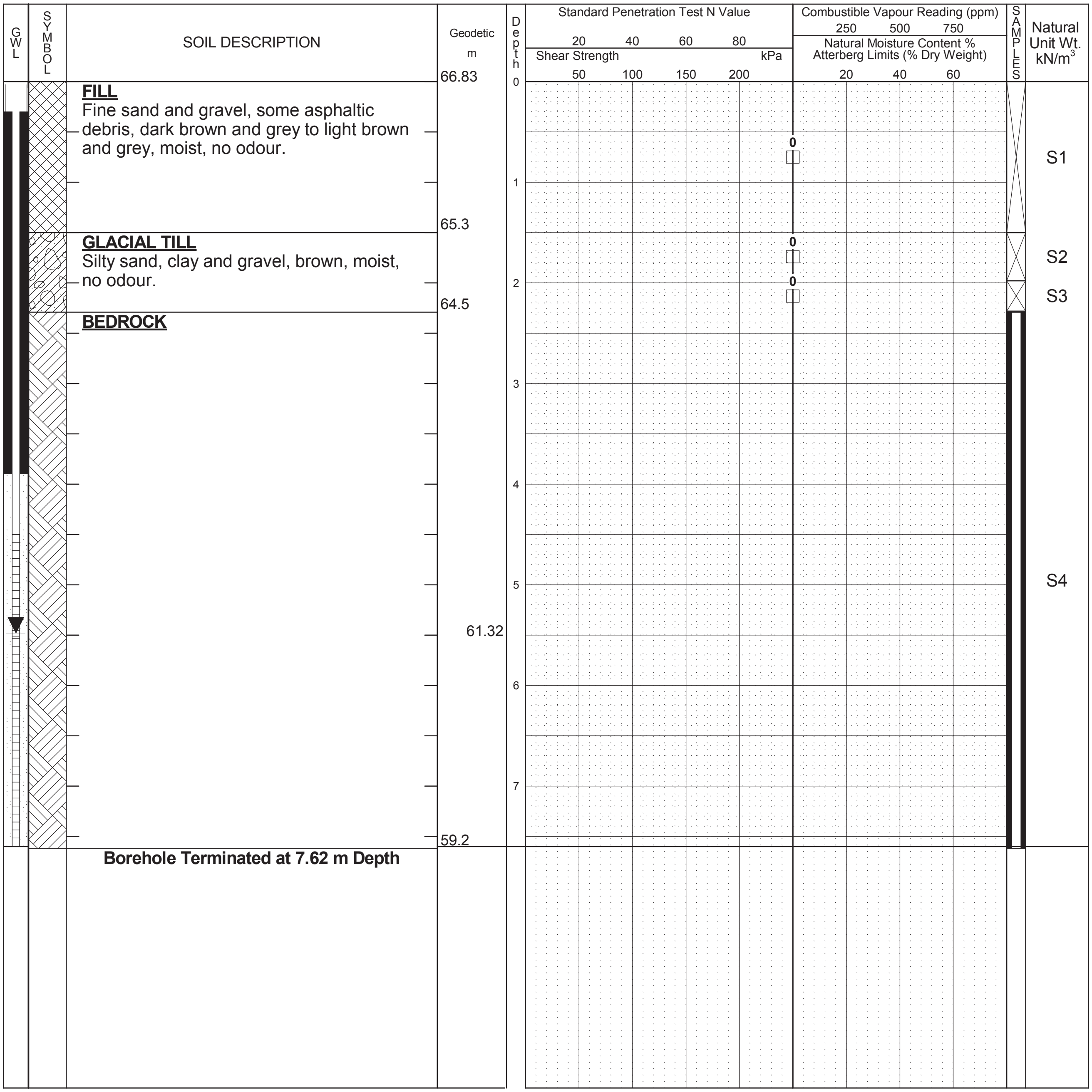
Date Drilled: October 6th, 2017

Drill Type: Geoprobe 420M

Datum: Geodetic

Logged by: JO Checked by: MGM

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



Geodetic m	Soil Description	Samples
66.83	**FILL** Fine sand and gravel, some asphaltic debris, dark brown and grey to light brown and grey, moist, no odour.	S1
65.3	**GLACIAL TILL** Silty sand, clay and gravel, brown, moist, no odour.	S2 S3
64.5	**BEDROCK**	S4
59.2	**Borehole Terminated at 7.62 m Depth**	

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. A flushmount monitoring well with a 32 mm slotted standpipe was installed in the borehole upon completion.
3. Field work was supervised by an exp representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241482-B0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
17 days	3.8	
24 days	5.5	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 32696

Report Date: 20-Aug-2021

Order Date: 17-Aug-2021

Project Description: PG5946

	Client ID:	BH1-21 SS4	-	-	-
	Sample Date:	06-Aug-21 09:00	-	-	-
	Sample ID:	2134258-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	90.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.92	-	-	-
Resistivity	0.10 Ohm.m	55.0	-	-	-
Anions					
Chloride	5 ug/g dry	32	-	-	-
Sulphate	5 ug/g dry	34	-	-	-

APPENDIX 2

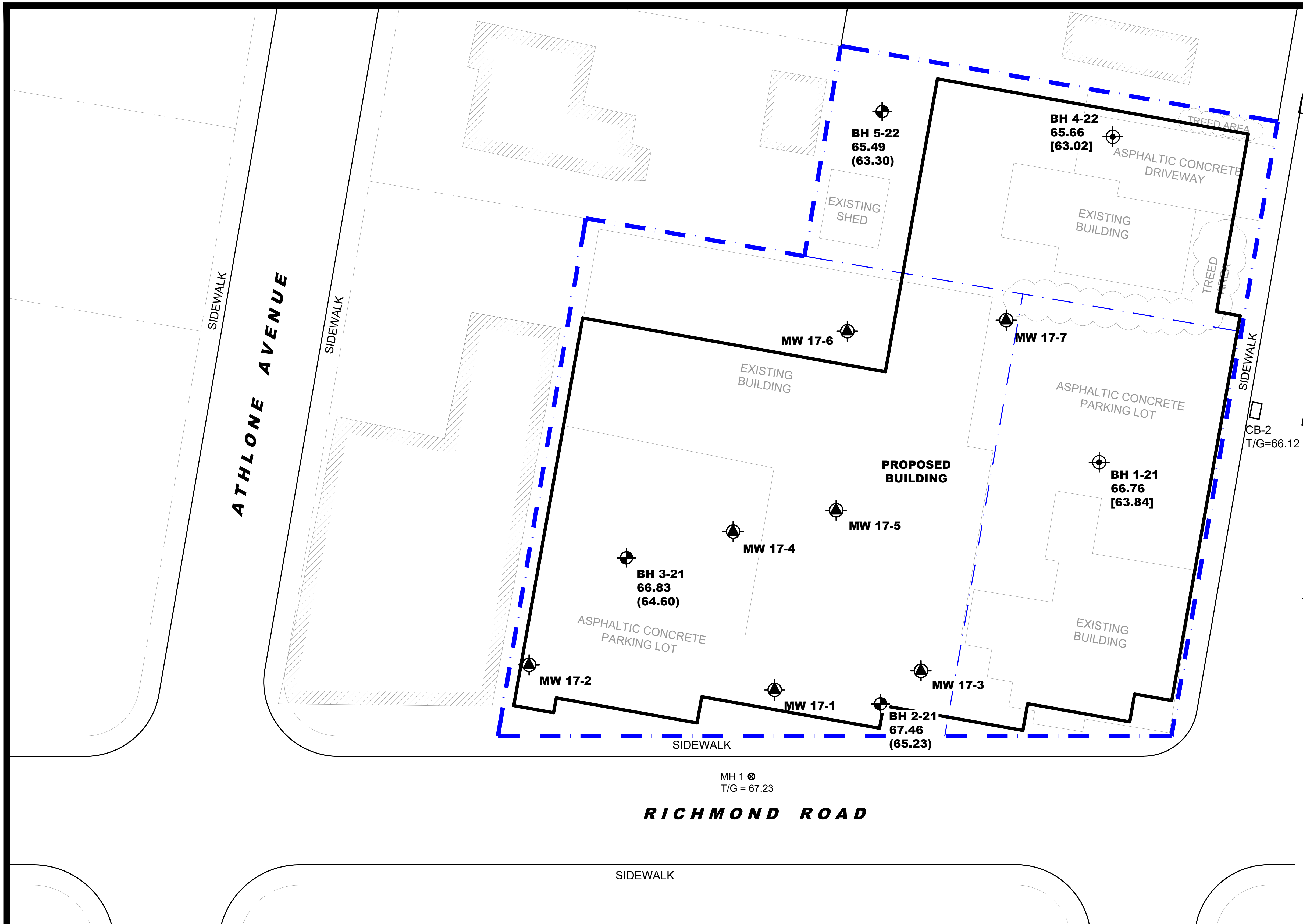
FIGURE 1 – KEY PLAN

DRAWING PG5946-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

⊕	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
▲	MONITORING WELL (BY OTHERS)
66.76	GROUND SURFACE ELEVATION (m)
[63.84]	BEDROCK SURFACE ELEVATION (m)
(65.23)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY VINCENT P. COLIZZA ARCHITECT INC.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

PATERSON GROUP
SOLUTION ORIENTED ENGINEERING

NO.	REVISIONS	DATE	INITIAL
1	ADDED 2022 BOREHOLES	28/07/2022	NS

2828727 ONTARIO INC. C/O YOUNG STREET CAPITAL
GEOTECHNICAL INVESTIGATION
PROPOSED REDEVELOPMENT
OTTAWA, 249, 251, 255 RICHMOND ROAD & 372 TWEEDSMUIR AVENUE ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:300	Date: 08/2021
Drawn by: JM	Report No.: PG5946-1
Checked by: MS	Dwg. No.: PG5946-1
Approved by: DJG	Revision No.: 1

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11x17